EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS
Earthquake Engng. Struct. Dyn. **28**, 479–500 (1999)

EFFECTS OF STRUCTURAL WALLS ON THE ELASTIC–PLASTIC EARTHQUAKE RESPONSES OF FRAME–WALL BUILDINGS

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SUMMARY

Effects of structural walls on the elastic–plastic earthquake response of short- to medium-height reinforced concrete buildings were investigated. The analytical model consists of independent lumped mass systems representing walls and frames connected at each floor. The wall structure undergoes flexural as well as shear deformation and fails in shear at relatively small story drifts, the frames deforming only in shear. As a measure of structural damage, the ductility factor responses of frame structures were calculated for different combinations of base shear coefficients for the frames and walls. In buildings with relatively weak frames, the installation of structural walls did not improve the large plastic response of the frames up to the point where the walls were unfailed in shear and the ductility factors of the frame structure were suddenly reduced to a very small number. For relatively strong frames, however, the response displacements decreased gradually as the number of walls increased, whether or not the walls failed. Empirical formulas for the required base shear coefficients of the walls and frames which gave a target ductility factor response also were derived for two particular groups of accelerograms. These equations should be of practical use in designing frame-wall type buildings and in retrofitting damaged buildings. Copyright © 1999 John Wiley & Sons, Ltd.

KEY WORDS: frame–wall buildings; inelastic response; ductility factor; base shear coefficient

INTRODUCTION

Reinforced concrete buildings with structural walls have shown superior seismic performance in the earthquakes of the last 30 years, particularly rigid frame-type buildings. Experiences from past earthquakes clearly indicate that the installation of structural walls may increase the overall rigidity of buildings, thereby reducing seismic distortion. Despite the known superiority of frame–wall-type structures, until the 1980s, engineers and researchers in New Zealand, Japan, the United States and other countries, gave preference to ductile frame structures, mainly because of their relatively poor understanding of this hybrid structure consisting of walls and frames.

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CCC 0098–8847/99/050479–22$17·50

Received 10 July 1998
Revised 23 October 1998

Only recently has a large body of experimental knowledge about frame–wall structures been accumulated. Most of the research published on the seismic behaviour of reinforced concrete walls has addressed two topics; ultimate shear strength[1–6] and criteria for designing walls against shear forces.[7,8] Much effort has been made to prevent structural walls from failing in the brittle shear mode; but it is also important to obtain a better understanding of the shear failure mode common to the shear walls of short buildings. In this study, we treated short- to medium- height buildings with walls having small shear–span ratio. No particular attention was paid to mitigating undesirable brittle shear failure.

Research on the effects of walls on the seismic response of reinforced concrete frame–wall structures has consisted of the investigation of the performance of frame-wall structures during past earthquakes,[9–11] experimental investigations,[12–14] and analytical investigations.[14,15] Integrated studies of the relation between the number of shear walls and the overall response of frame-wall buildings have been limited. Single-degree-of-freedom systems have been used to evaluate the combinations of frame and wall shear capacities required to yield a specified response value, particularly in connection with the retrofitting of and strengthening plans for damaged buildings.[16]

By introducing multi-degree-of-freedom systems to independently represent the wall and frame structures, we could investigate the effects of the number of structural walls on the elastic–plastic response of short- to medium-height frame–wall buildings by using extensive numerical analyses. Shear modes of failure in structural walls, common to short- to medium-height buildings, were emphasized throughout the study, special interest being placed on the behaviour of frame structures after the break down of structural walls. The required combinations of frame and wall shear capacities in a frame–wall structure which gives a specified amount of response are discussed. The results presented in the paper apply mostly to the existing buildings which have not been designed following the modern capacity design procedure.

ANALYSIS PROCEDURE AND INPUT GROUND MOTIONS

To determine the earthquake response of reinforced concrete frame–wall structures, we developed a computer code that analyses the elastic–plastic behaviour of planer structures. Weights of the building were lumped at floor levels, and the horizontal displacements of the frame and wall structures were set equal on each floor (Figure 1). The base was assumed to be fixed for both the frames and walls. The sectional properties of walls were assumed uniformly distributed along the height of the building. Geometrical non-linearities, e.g., P–Δ effects, were ignored.

Modelling of frame structures

The frame was modelled as a series of shear springs, the system representing a column failure mechanism (Figure 1). The skeleton curve for the story shear force versus the story drift relationship was idealized as a tri-linear curve using the Takeda hysteresis model (Figure 4 (a)).[17–20] The characteristic values used to define the skeleton curves were fixed (Figure 2) throughout the study to avoid unnecessary complexity. The story rotational angle at yielding was set at 1/150.[21,22]

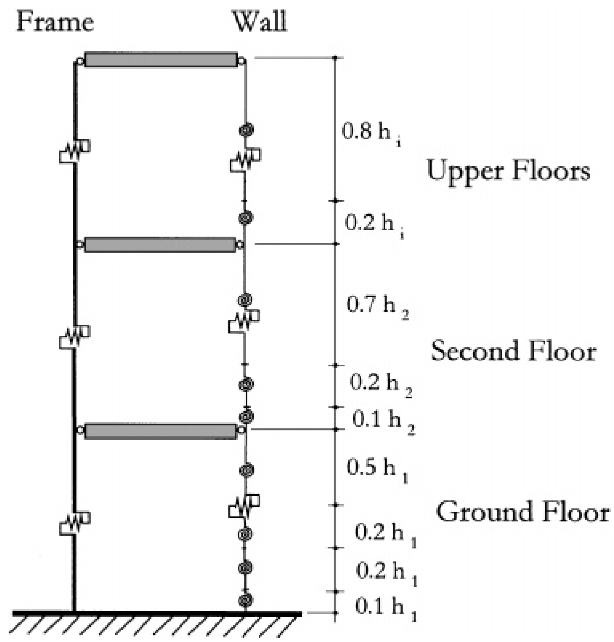


Figure 1. Analytical model of frame–wall structures: Frames and walls are treated as independent series of springs and are connected at each floor. Frames consist of a shear spring on each floor, and walls of a shear spring and a few rotational springs. Masses are concentrated at floor level

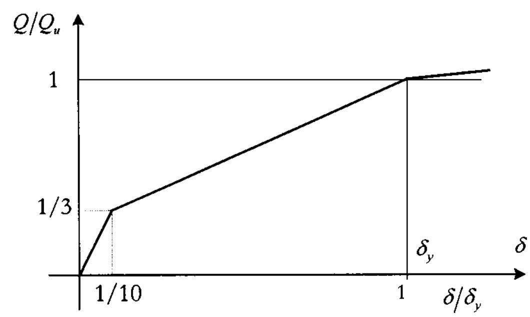


Figure 2. Skeleton curves for frames. The drift angle at yielding was set at 1/150 for every floor

Modelling of wall structures

The lumped spring model of structural walls developed by Takayanagi[23] (1976) was adopted to account for the inelastic flexural behaviour of the walls. This model, in which the wall members are divided into several sub-elements, distributes inelastic flexural deformation into finite segments of a wall member. The stress resultants at the centroid of the sub-elements were used to control non-linear flexural deformation of the sub-elements. Four, three, and two sub-divisions were made for the ground, second, and upper storys because large inelastic deformation usually

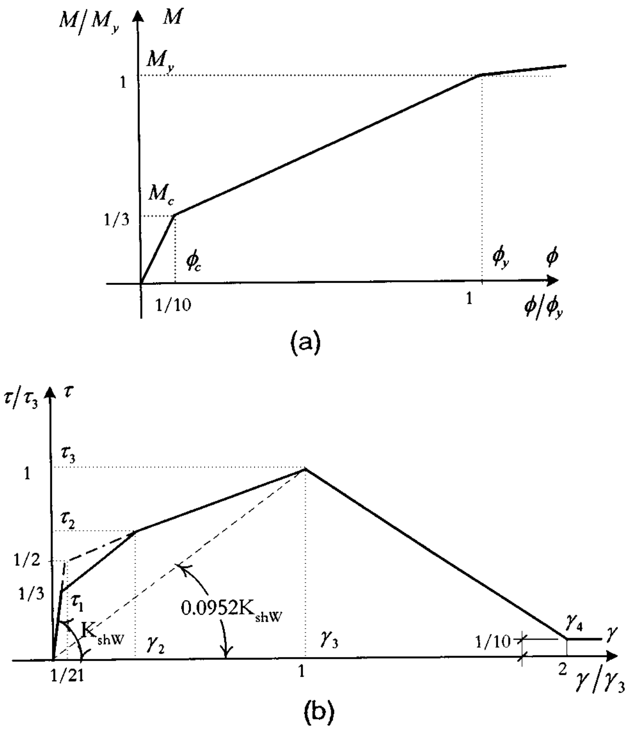


Figure 3. Skeleton curves for walls. The curvature at yielding was set at 0.3 per cent of the wall width. The drift angle at yielding in shear was set at 1/250 for every floor: (a) moment–curvature relationship; (b) stress–strain relation in shear

occurs only in the lower storys (Figure 1). The shear deformation of the walls was represented by the horizontal shear springs.

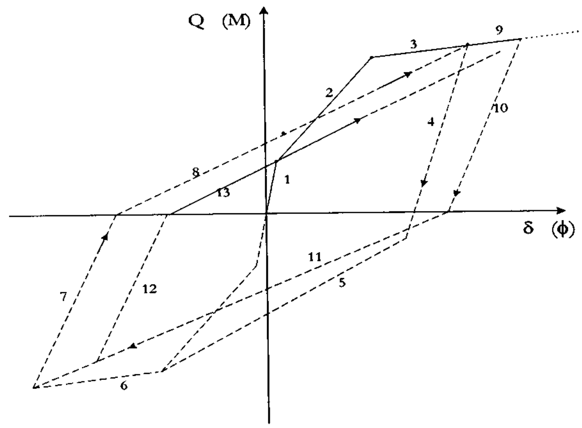
The bending moment versus curvature relationship was idealized as a tri-linear curve (Figure 3 (a)). For rectangular and symmetrically flanged walls, a conservative estimation for the yielding curvature, ϕ_y, was made[24]

$$\phi_y \approx \frac{0{\cdot}003}{L_W} \tag{1}$$

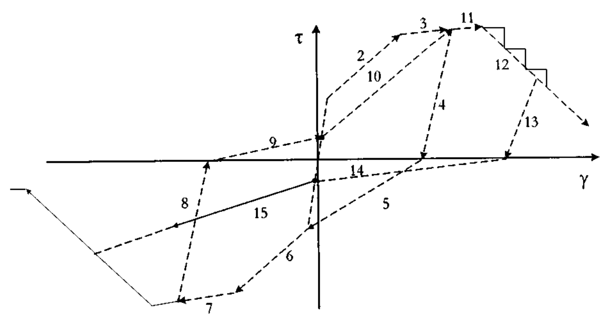
where L_W is the wall width (Eq. 2).

The Takeda model was used to simulate the hysteretic flexural behaviour of each sub-element (Figure 4(a)).

The skeleton curve for the shear stress versus shear strain relationship of the wall members was idealized as a four-linear curve (Figure 3(b)). The shear deformation of the wall at the ultimate stage, γ_3, was set constant at 1/250.[5,21,23] To simulate the shear failure of the wall, a progressive failure mode was introduced, because if the wall resistance disappears suddenly at failure an extremely large unbalanced force will act as an impulsive load on the rest of the building. The hysteresis rule proposed by Vallenas,[3] modified to include the negative slope of the skeleton curve of the shear stress–shear strain relationship, was used to simulate the shear behaviour of the wall (Fig. 4(b)).



(a)



(b)

Figure 4. Hysteresis rules adopted for the shear and rotational springs. The Takeda model was used for both the shear deformation of the frames and bending deformation of the walls. The Vallenas model, modified to include negative skeleton curves, was used for the shear deformation of the walls. (a) takeda model; (b) modified Vallenas model

Time integration scheme

The direct integration method was used to solve the equation of motions. The numerical method utilized two assumptions to linearize the relations among the time response functions: First, the incremental forces and displacements were related linearly by instantaneous stiffness over a short time. Second, the acceleration changed linearly within a short period. The damping matrix was expressed as a linear combination of stiffness and mass matrix, where the first and second mode damping factors were chosen as 0·05. The time interval chosen for integrating the equation of motions was 0·005 s.

To deal with negative slope in the shear force–shear displacement relation of the walls, incremental displacements were calculated for zero stiffness and the negative change of forces then treated as the applied loads in the next step.

Table I. Accelerograms used for input ground motions. A_{max}: peak ground accelerations

ID	Station	Direction	Earthquake	Amax(cm/s^2)
SLM	Sylmar	E–W	1994, Northridge	826·7
TAFT	Taft	E–W	1952, Kern County	175·9
ELC	El Centro	N–S	1940, Imperial Valley	341·3
KJMA	Kobe J. M. A	N–S	1995, Hyogoken-nanbu	818·0
HACH	Hachinohe	E–W	1968, Tokachi-oki	181·7
FKI	Osaka Gas Fukiai Station	N–S	1995, Hyogoken-nanbu	802·0
TOH	Tohoku Unversity	N–S	1987, Miyagiken-oki	258·2

Input ground motions

Seven typical strong ground motion records were used as inputs in the response analyses (Table I). The time duration for each record was set at 30 s, except for that of the Hachinohe record which was 50 s. Ground motions were normalized to have the same peak ground acceleration, 0·8g, representing the peak ground accelerations recorded in the most violently shaken areas in Kobe (1995) and in Northridge (1994). An additional peak ground acceleration value, 0·5g, was adopted to represent ground motions of moderate intensities.

Parameters and characteristic values

For easier interpretation of the numerical results, we reduced the number of parameters which describe the character of frame–wall buildings to as few as possible.

(1) Building Height:
Frame–wall structures with four, eight, and twelve floors were analysed as representative of short- to medium-height buildings. The floor heights of the buildings were set constant at 3·5 m in all cases.

(2) Base shear coefficient of frames:
To express the bearing capacity of the frame structures whose responses were used to measure the total damage to structures, the base shear coefficient of the frames, C_{0F}, was introduced. Variations in yielding strength and stiffness along the height of the buildings were assigned by the following procedures: (1) yielding strength was distributed over the stories so that all would yield simultaneously under the static design earthquake forces specified in the Japanese Design Code;[26] (2) distribution of the initial elastic stiffness over the stories was determined so that all would undergo the same yield deflection (drift angle at yielding was set at 1/150) under the specified design load. By this method, the stiffness of the frame was proportional to the yielding strength, and the natural periods became anti-proportional to the square root of the base shear coefficients. The overall image of the natural periods of frame structures with different base shear coefficients is given in Table II.

(3) Base shear coefficient of the walls:
The number of walls in a frame–wall building is the fundamental parameter for discussing the effects of walls on the response of frame–wall buildings. For convenience, the base shear

Table II. Comparative natural periods, T, of frame structures with different base shear coefficients, C_{0F}

Four-story building		Eight-story building		Twelve-story building	
C_{0F}	T(s)	C_{0F}	T(s)	C_{0F}	T(s)
0·3	0·49	0·2	0·80	0·1	1·35
0·4	0·43	0·3	0·65	0·2	0·96
0·5	0·38	0·4	0·56	0·3	0·78
0·6	0·35	0·5	0·51	0·4	0·68

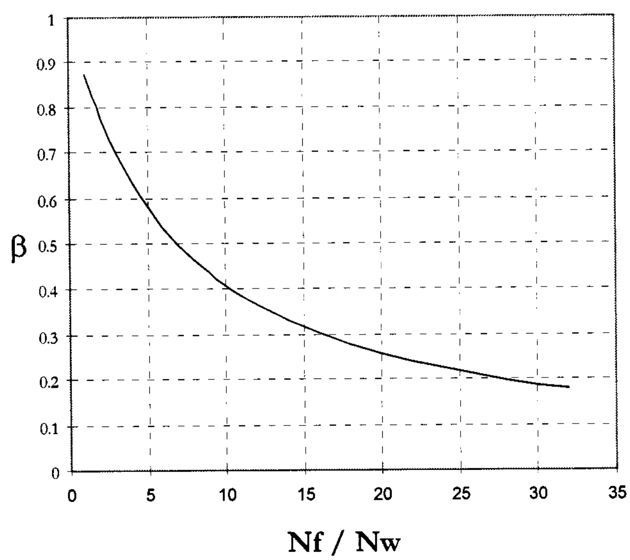


Figure 5. Relevant range of the wall ratio, β, in terms of the ratio of the numbers of frames to walls, N_f/N_w. This relationship was derived using the following assumptions: (1) the wall had a rectangular cross-section 5 m long, 0·2 m thick, and a shear stress capacity of 230 Tf/m^2, [5,6] (2) the column of the frame structure had cross-sectional dimensions of 0·6 × 0·6 m and a shear stress capacity of 100 Tf/m^2 [6]

coefficient of the wall structures was expressed as a ratio to the total base shear coefficient of the building;

$$\beta = \frac{C_{0W}}{C_{0F} + C_{0W}}$$

where C_{0W} and C_{0F} are the respective base shear coefficients of the walls and frames defined as yield strength divided by the total weight of the building.

To obtain an image of the relevant range of β in actual frame–wall buildings, an example of this parameter was plotted against the number of frames per single wall (Figure 5). The range of

β from 0·4 to 0·85 was considered relevant because the number of frames per wall, N_f/N_w, which corresponds to these values varies from 1 to 10, that was common to the frame–wall buildings considered.

Another important factor for controlling the earthquake response of structural walls is the height–width ratio, or put more physically, the ratio of the flexural and shear deformation of the walls. The flexural ratio, ρ, is defined as the ratio of shear and flexural displacements at the top of walls under a unit horizontal load;

$$\rho = \frac{3EI}{GAH^2}$$

where EI is the flexure rigidity, GA the shear rigidity of the wall cross-sections, and H the height of the building. The relevant value of ρ for the buildings considered is about 0·2. Moreover, preliminary case studies showed that the response displacements did not vary much (within 20 per cent) for the range $\rho = 0{\cdot}2$–$0{\cdot}5$. We therefore used $\rho = 0{\cdot}2$ throughout the study for simplicity and a more comprehensive interpretation of the numerical results.

From the definition of the flexural ratio, ρ, and by assuming the cross-section of the walls as a rectangular one, the wall length was determined as follows:

$$L_W = H\sqrt{\rho\,\frac{4G}{E}} \tag{2}$$

where H is the total height of the building, and E and G are, respectively, Young modulus and shear modulus of the wall.

The shear rigidity, K_{shW}, the flexural rigidity, K_{flW}, and the yielding moment, M_y, of the wall at the base were defined as follows:

$$K_{shW} = \frac{C_{0W}}{\gamma_3 h_s 0{\cdot}0952}$$

$$K_{flW} = \tfrac{1}{3}\rho K_{shW} h_s H^2$$

$$M_y = 0{\cdot}3 K_{flW}\phi_y$$

where h_s is the story height of the building, γ_3 the shear deformation of the wall at the ultimate stage, and ϕ_y the yielding curvature (equation (1)). It is noted that all these quantities were normalized by the total weight of the building.

Examples of natural periods of frame–wall buildings with regard to wall ratio, β, are shown in Figure 6.

EFFECTS OF THE NUMBER OF WALLS ON THE ELASTIC–PLASTIC RESPONSE OF FRAME–WALL BUILDINGS

Studies of the earthquake response of frame–wall buildings after their walls fail and determinations of the required plastic deformation capacities of the frames are of particular practical importance for avoiding the complete collapse of buildings. The maximum frame response ductility factor, μ, was introduced as a measure of structural damage to frame–wall buildings at

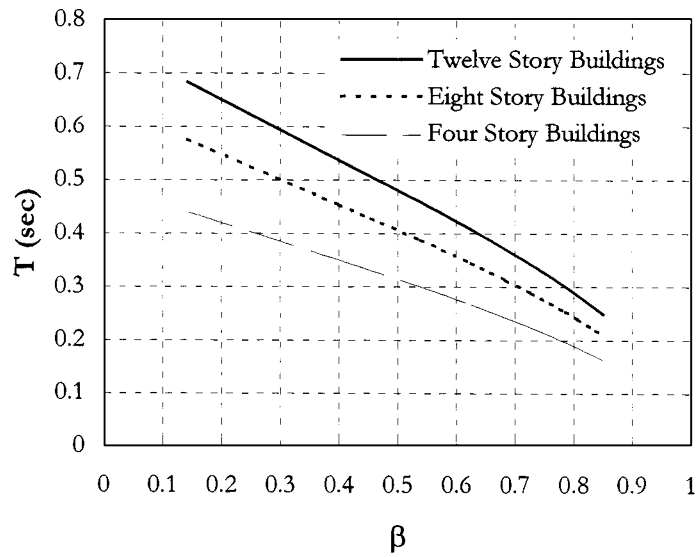


Figure 6. Natural periods, T, of frame–wall buildings having different wall ratios β. The base shear coefficient of the frames were set at $C_{0F} = 0{\cdot}3$

the end of strong ground motions:

$$\mu = \mathrm{Max}\left(\frac{\delta_{i\max}}{\delta_{iFy}}\right)$$

where $\delta_{i\max}$ and δ_{iFy} are the maximum and yielding displacements in the ith story of the frame structures.

Ductility factors, μ, for four-, eight- and twelve-story buildings whose frames have the same base shear coefficient ($C_{0F} = 0{\cdot}3$) are plotted in Figure 7(a)–(c) with respect to β, for all the input ground motions. The peak ground accelerations were set equal to $0{\cdot}8\,g$ in the following. Strong ground motions, according to their damage potentials, are roughly divisible into two groups: the Hachinohe, Fukiai, and the Tohoku University accelerograms which produce large displacement responses, group 1; and the El Centro, Kobe J.M.A (Japan Meteorological Agency), and Sylmar accelerograms, group 2. The grouping becomes less distinct for taller buildings.

The response ductility factors for the Hachinohe and Sylmar accelerograms are shown in Figures 8 and 9 as examples of the $\mu - \beta$ relations for frames with different base shear coefficients.

For frame-wall buildings with relatively weak frames ($C_{0F} < 0{\cdot}5$ when subjected to the first group of strong ground motions) shear failure of the walls, indicated by open circles, resulted in large displacement responses and even in failure of the structure. An increase in the number of walls sometimes worsened the response of frame–wall buildings. The larger the number of walls, the larger the impact of the failure of the walls on the frame which, due to the sudden loss of shear resistance, may undergo a large plastic deformation. This feature was common to all the short- and medium-height buildings considered. Once the wall became unfailed the response decreased markedly. In these cases, the walls carried the main part of the earthquake forces and greatly reduced deformation of the total structure. For example, in the case of the Hachinohe

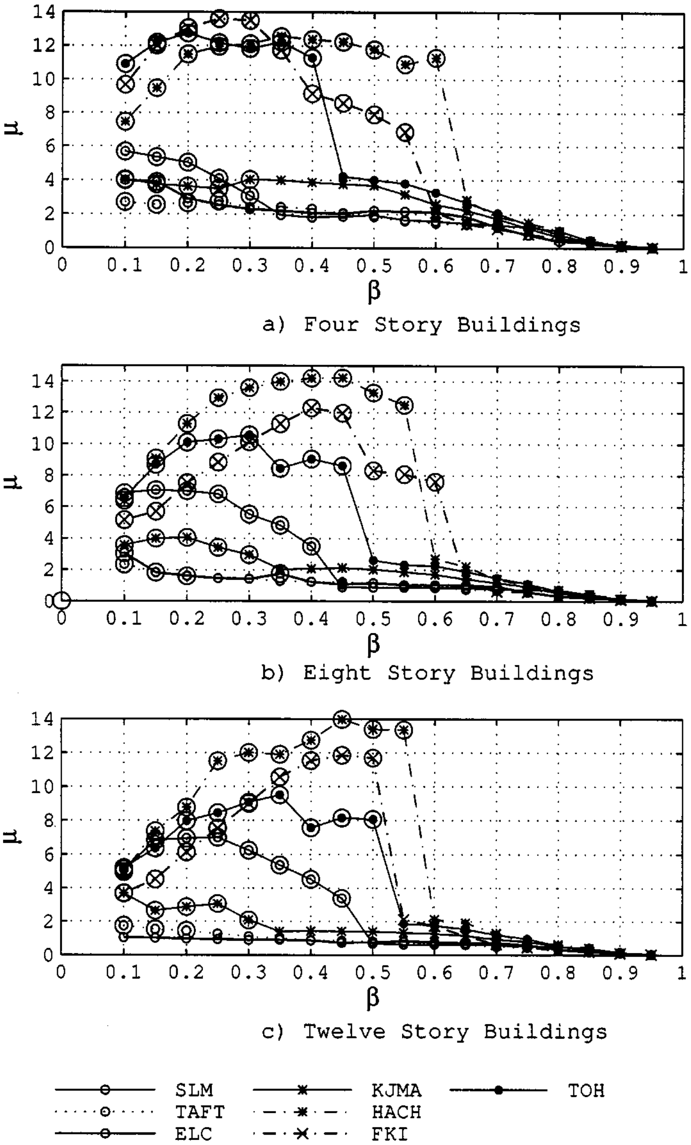


Figure 7. Relation between the ductility factor response, μ, and the wall ratio, β for (a) four-; (b) eight-, and (c) twelve-story buildings. Large open circles indicate that the walls have already failed in shear. Peak ground accelerations of all the accelerograms were kept equal at $0{\cdot}8g$, and the base shear coefficients of the frames were set at $C_{\mathrm{OF}} = 0{\cdot}3$

accelerogram (Figure 8(a)), by increasing β from 0·10 to 0·35, the maximum ductility factor increased by about 66 per cent, whereas by increasing β from 0·6 to 0·65 the response of the structure decreased from a very large ($\mu = 11{\cdot}2$) to a moderate ($\mu = 2{\cdot}8$) plastic deformation.

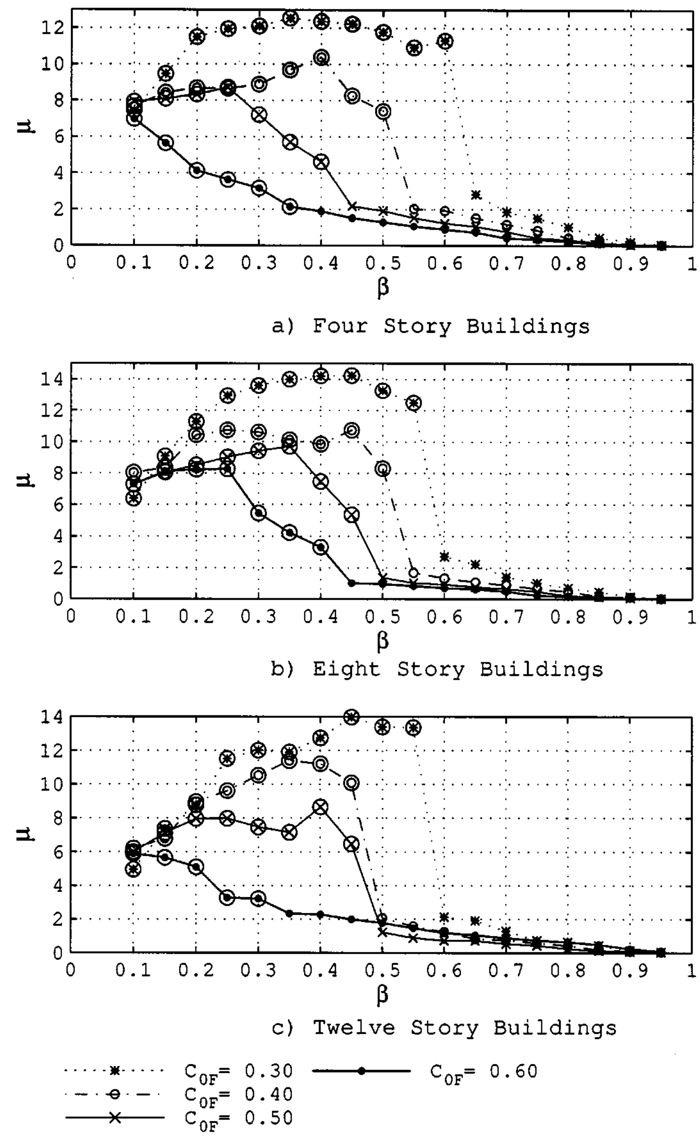


Figure 8. Relation between the ductility factor, μ, and wall ratio, β, for: (a) four-; (b) eight-; and (c) twelve-story buildings with different base shear coefficients for their frames. Large open circles indicate that the walls have already failed in shear. The Hachinohe accelerogram, with a peak ground acceleration of $0{\cdot}8g$, was used as the input ground motion

In short frame–wall buildings with relatively strong frames ($C_{0F} > 0{\cdot}5$ when subjected to the first group of ground motions), and an increase in the number of walls improved the response whether or not the wall failed. In such cases the frame was strong enough to resist the impact of the sudden loss of wall resistance and subsequent earthquake actions.

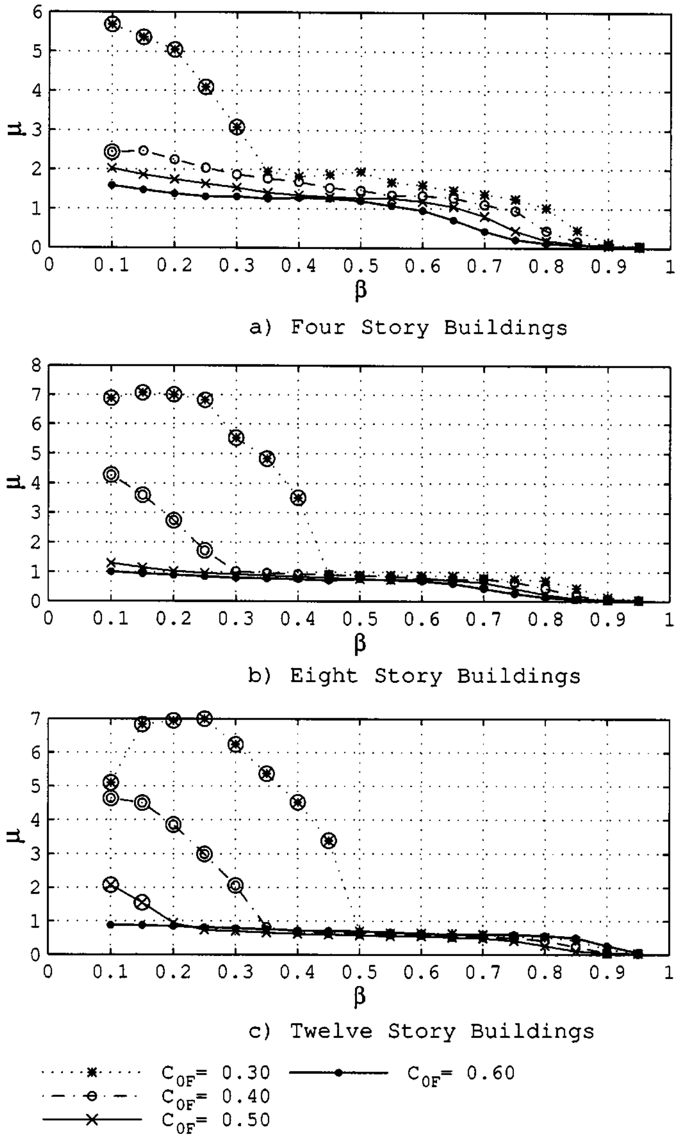


Figure 9. Relation between the ductility factor, μ, and the wall ratio, β, for (a) four-story; (b) eight-story; and (c) twelve-story buildings with different base shear coefficients of their frames. Large open circles indicate that the walls have already failed in shear. The Sylmar accelerogram with the peak ground acceleration of $0{\cdot}8g$ was used as the input ground motion

THE NUMBER OF WALLS NEEDED FOR SPECIFIED PERFORMANCES

The wall and frame base shear capacities were changed systematically to trace all possible combinations for which the maximum displacement responses reach specific target values. Two

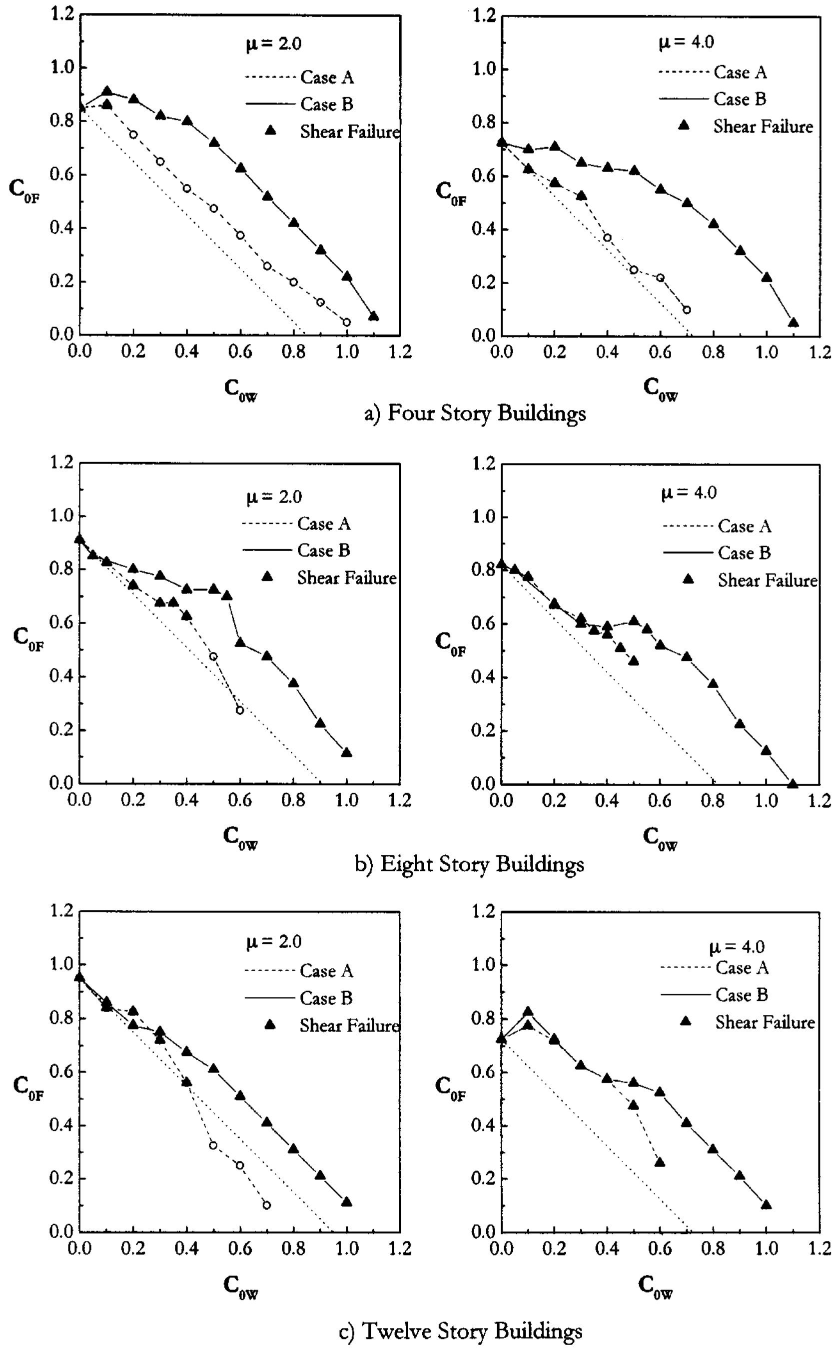


Figure 10. The required combinations of base shear coefficients for frames and walls that resulted in the specified target responses, $\mu = 2$ (left side) and $\mu = 4$ (right side). Broken and solid lines correspond, respectively, to cases A and B in which inelastic and elastic flexural deformation were allowed for wall structures. The Hachinohe accelerogram with peak ground acceleration of $0{\cdot}8g$ was used

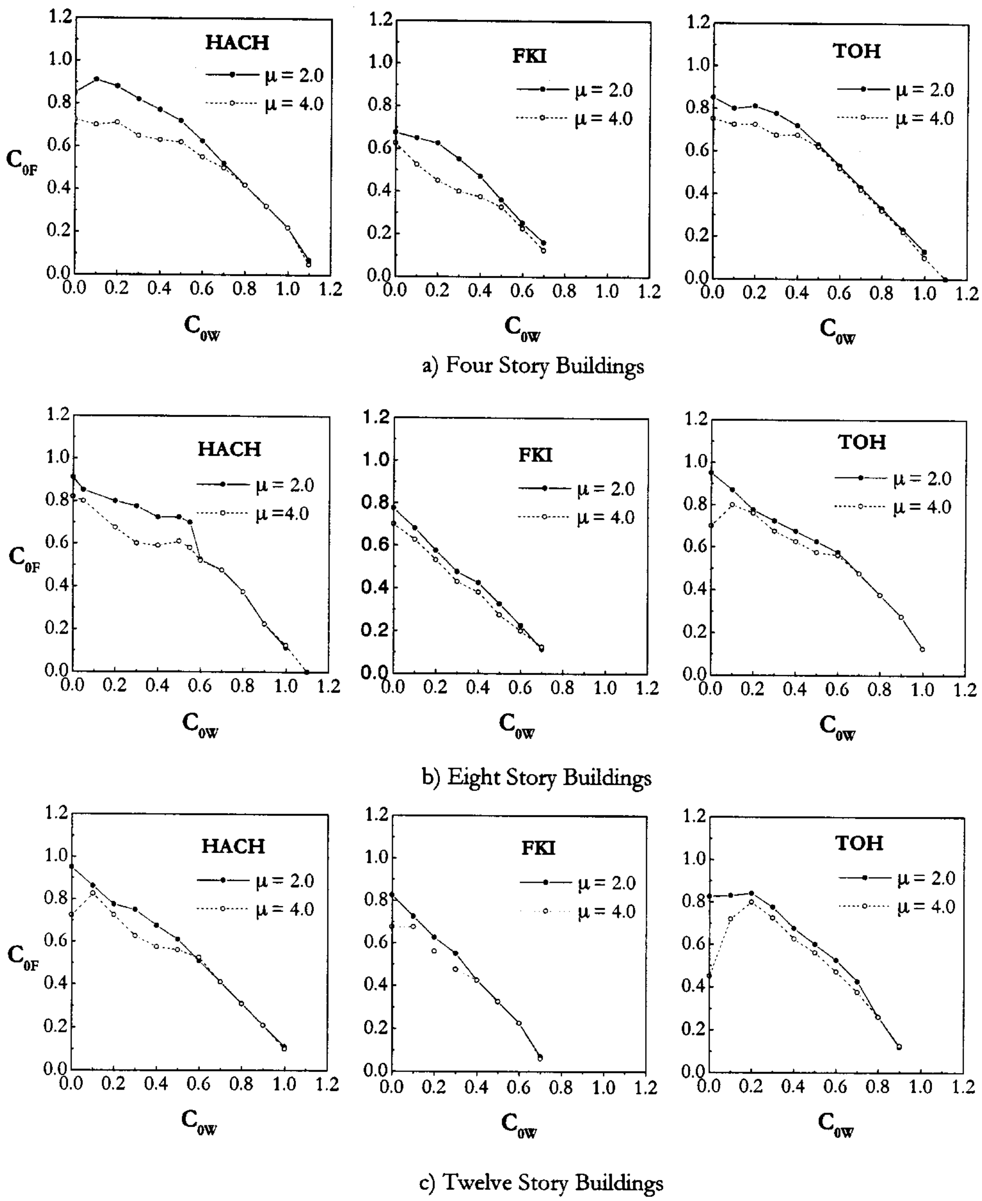


Figure 11. Comparative base shear coefficients for frames and walls which gave the two target ductility factors $\mu = 2$ and 4. The first group of accelerograms (Hachinohe, Fukiai, and Tohoku University) with the peak ground acceleration of $0{\cdot}8g$ was used

levels of the ductility factor response of the frames, $\mu = 2$ and 4, were selected as the target displacement responses.

In our analyses of structural walls, no limitation was placed on plastic flexural deformation, and in some cases, when the walls did not fail in shear, extremely large flexural deformation may

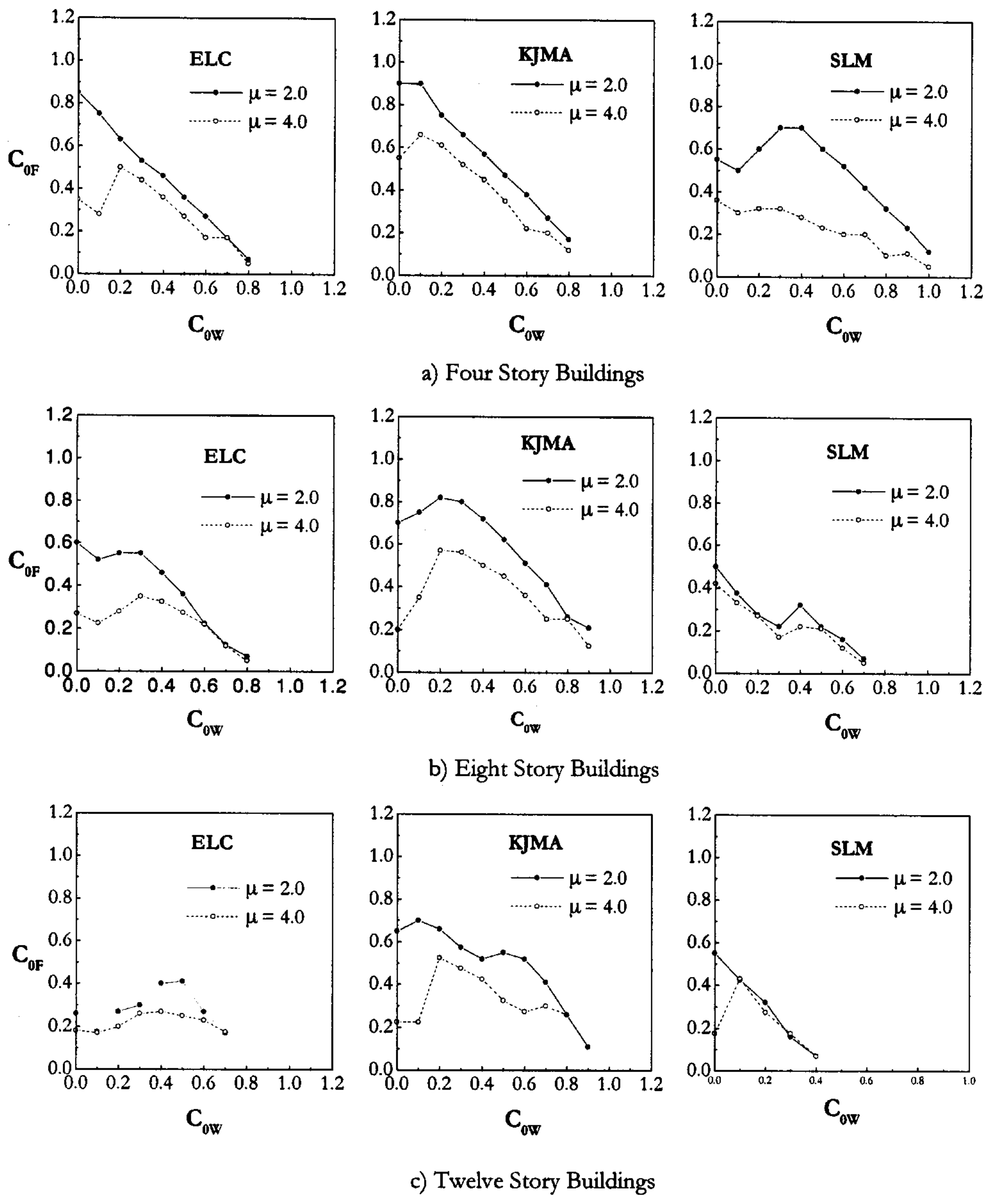
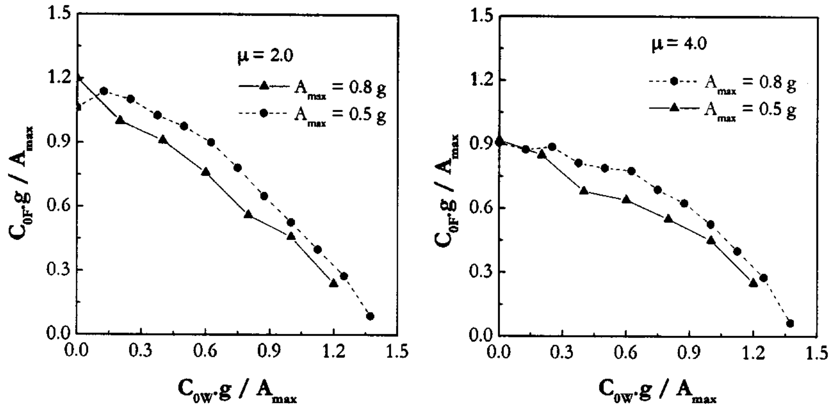
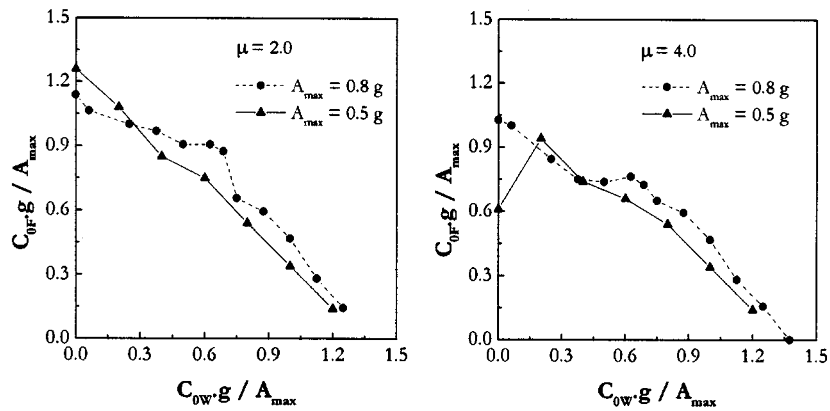


Figure 12. Comparative base shear coefficients for frames and walls which gave the two target ductility factors $\mu = 2$ and 4. The second group of accelerograms (El Centro, Kobe Meteorological Observatory, and Sylmar) with the peak ground acceleration of $0{\cdot}8g$ was used

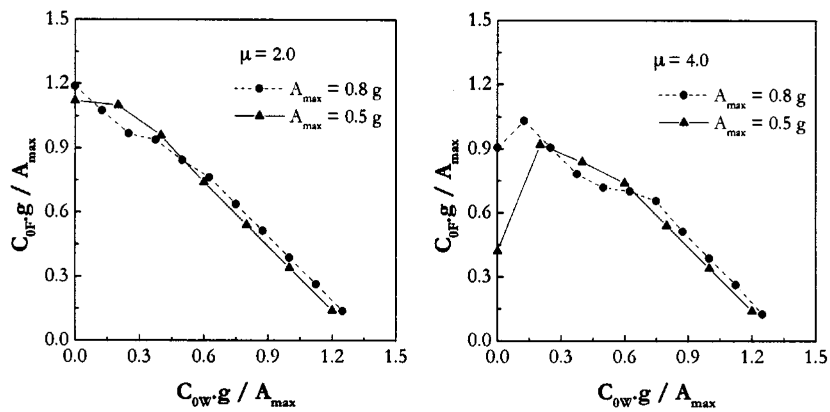
have occurred (Figures 7–9). To exclude such unrealistic situations, an extreme condition was introduced such that flexural deformation of walls always remained elastic (denoted by Case B compared with the original analysis Case A).



a) Four Story Buildings



b) Eight Story Buildings



c) Twelve Story Buildings

Figure 13. The required combinations of base shear strength for frames and walls to produce the target ductility factors $\mu = 2$ (left side) and $\mu = 4$ (right side) are shown for: (a) four-; (b) eight-; and (c) twelve-story buildings. Yielding accelerations of the frames and walls were normalized by the peak ground accelerations. Results for the Hachinohe accelerogram with two levels of PGA, $A_{max} = 0{\cdot}8g$ and $A_{max} = 0{\cdot}5g$, are compared

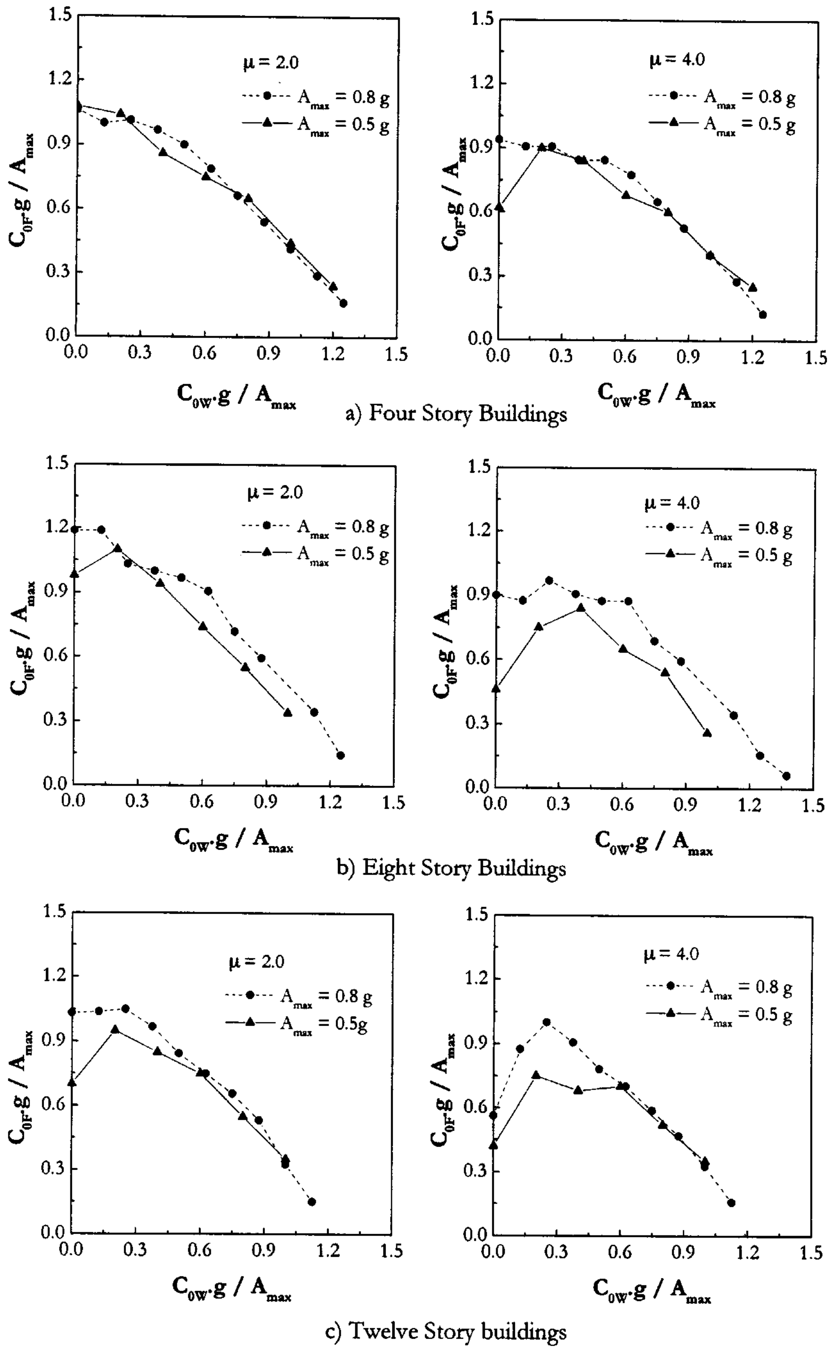


Figure 14. The required combinations of base shear strength for frames and walls to produce the target ductility factors $\mu = 2$ (left side) and $\mu = 4$ (right side) are shown for (a) four-, (b) eight- and (c) twelve-story buildings. The yielding accelerations of the frames and walls were normalized by the peak ground accelerations. Results for the Tohoku University accelerogram with two levels of PGA, $A_{max} = 0{\cdot}8g$, and $A_{max} = 0{\cdot}5g$, are compared

Figure 10(a)–(c) shows the required wall and frame base shear coefficients of four- eight- and twelve-story buildings subjected to the Hachinohe accelerogram. The required total base shear coefficients of frame–wall buildings in Case A always is smaller than that in Case B because a part of the energy transferred to the structure is absorbed by the flexural plastic deformation of the walls in Case A; whereas, the walls always fail in shear in Case B, and the energy absorbed by them is much smaller. Differences between the two cases increased with increasing β as the relative affect of the walls in frame–wall buildings increased.

Interestingly, shear failure did not always occur in Case A, and the required total base shear coefficients in some cases were even smaller than the value for a pure frame structure. Figures 11 and 12 show the required base shear coefficients of walls and frames for the first and second groups of accelerograms. Case B is discussed exclusively in the following.

In the range of the small base shear coefficients of the walls, the required base shear coefficients of the frames became bounded, and the contour lines for the large target value, $\mu = 4$, were slightly less than those for smaller target value, $\mu = 2$. In the region of the large base shear coefficients of the walls, the contour lines were very close to each other and could be approximated by a straight line. The response of the structures changed drastically from the elastic range to very large plastic displacements once the total of the base shear coefficients became slightly smaller than the required values. In other words, this contour line did not represent the base shear coefficients required for walls and frames to produce the target displacements, rather it represented the minimum strength required for the walls not to fail in shear. This was clearly observed for the first group of ground motions (Figure 11), whereas for the second group of accelerograms the contour lines were relatively flat for a wider range of wall base shear coefficients (Figure 12). As seen from Figure 10, the base shear coefficients required for frames and walls to produce a target response approached a straight line, representative of the constant total base shear coefficients, for a wider range of C_{0W} in Case A than in Case B.

In our analyses of frame structures, the initial stiffnesses depended on their base shear coefficients. In general, the base shear coefficients of the frames and walls could not be normalized by the peak ground accelerations to give the same ductility factor responses. According to the current concept of the performance-based design of buildings, the magnitude of seismic loads or the amplitude of the ground motions cannot be fixed at a single value, rather it varies for different levels of demanded performance. For convenience, it was considered best to redefine the required base shear coefficients of the walls and frames using the ratio to the peak ground acceleration:

shear strength ratio of the walls:

$$\alpha_{\mathrm{w}} = \frac{C_{0\mathrm{W}}g}{A_{\max}}$$

shear strength ratio of the frames:

$$\alpha_{\mathrm{F}} = \frac{C_{0\mathrm{F}}g}{A_{\max}}$$

The required base shear coefficients of the walls and frames were drawn in terms of the new variables, α_{W} and α_{F}, for the Hachinohe and the Tohoku University accelerograms with the peak ground accelerations of 0·5 and 0·8 g in Figures 13 and 14. The contour lines for the two levels of peak ground accelerations were close to each other, indicating that the required base shear strengths of the walls and frames were roughly proportional to the peak ground accelerations.

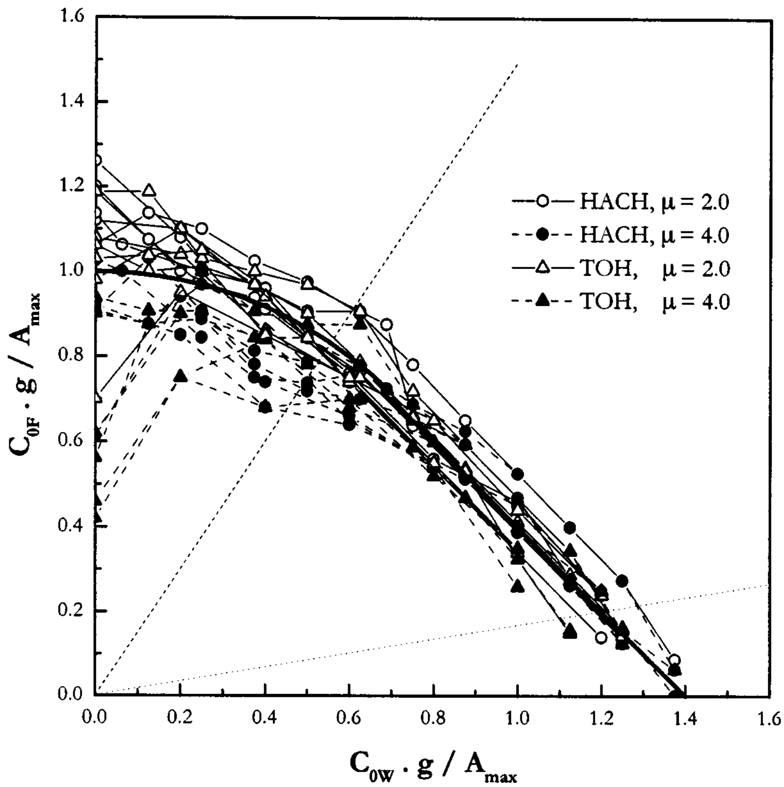


Figure 15. The required combinations of normalized base shear strength for frames and walls to produce the target ductility factors are shown for the Hachinohe and Tohoku University accelerograms. Results for different heights of buildings, different target values, and different peak ground accelerations were combined for this type of accelerogram. Bold lines show the empirical relations $\alpha_F + \alpha_W = 1{\cdot}4$ ($\alpha_F < \alpha_W$) and $\alpha_{F^2} + \alpha_{W^2} = 1$ ($\alpha_F > \alpha_W$) which gave the mean values of the normalized base shear strengths of the frames and walls. Dotted lines indicate the relevant range of the wall strength ratio; $0{\cdot}4 < \beta < 0{\cdot}85$

The contour lines of the required shear strength ratios of the walls and frames for the first group of accelerograms are plotted together in Figure 15, except for Fukiai which constantly gave smaller strength values than the Hachinohe and Tohoku University records. They were very close to each other even for different heights of buildings and different target response values. It should be noted that these numerical results were derived for Case B which always gave greater required strength ratios than Case A. The relevant region of α_W and α_F is shown by dotted lines which correspond to the relevant range of the wall strength ratio; $0{\cdot}4 < \beta < 0{\cdot}85$.

For convenience, the empirical formulas for estimating the required shear strength ratios of the walls, α_W, and the frames, α_F, were determined.

$$\alpha_W^2 + \alpha_F^2 = 1 \quad (\alpha_W < \alpha_F) \tag{3}$$

$$\alpha_W + \alpha_F = 1{\cdot}4 \quad (\alpha_W > \alpha_F) \tag{4}$$

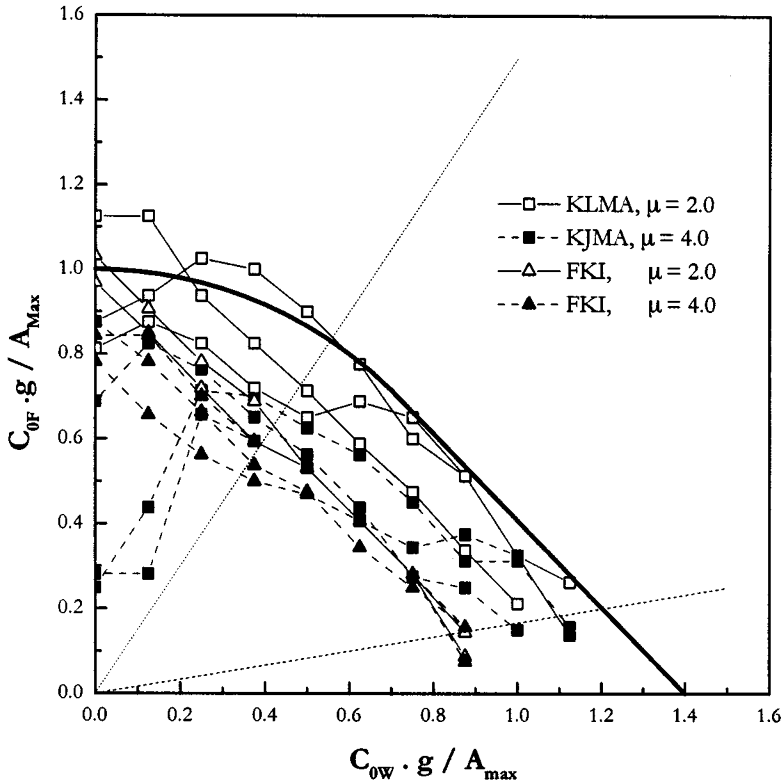


Figure 16. The normalized base shear strength for the Fukiai and Kobe J. M. A accelerograms were combined for different heights of buildings, two target ductility factors, and the peak ground acceleration, $A_{\max} = 0{\cdot}8g$. Bold lines show the empirical relations, equations (3) and (4), which gave an upper bound for this type of accelerogram. Dotted lines indicate the relevant range of the wall strength ratio; $0{\cdot}4 < \beta < 0{\cdot}85$

These relations express a circle and a straight line, as marked by the bold lines in Figure 15. They indicate average values of the required shear strength ratios of the walls and frames. The straight lines that express the constant total shear strength ratios of the walls and frames, as shown earlier, divided the elastic response and extremely large plastic displacements.

A similar chart for the Fukiai and Kobe, J. M. A. accelerograms is shown in Figure 16, in which expressions (3) and (4) give the upper bound of the required strength ratios. The contour lines fluctuate more widely than those in Figure 15, and the empirical formulas are reduced 15–20 per cent for the larger target value, $\mu = 4$.

The accelerograms used to draw Figure 15 were recorded in northern Japan during huge inter-plate earthquakes with magnitudes of about 8; whereas, those used in Figure 16 were recorded from the inland earthquake in Kobe of magnitude 7·2. As the required strength ratios for the Sylmar, El Centro and Taft accelerograms were smaller and more scattered than those in Figure 16; they were excluded from the present discussion.

SUMMARY AND CONCLUDING REMARKS

The effects of structural walls on the elastic–plastic response of frame–wall buildings are summarized as follows. When the base shear coefficients of frame structures were relatively small, i.e., $C_{0F} \leqslant 0{\cdot}5$ for the Hachinohe and Tohoku University accelerograms with $A_{max} = 0{\cdot}8g$, shear failure of the walls resulted in extremely large plastic displacements in the frame structures and even in failure of the total structure. The situation was not improved by increasing the number of walls until they were unfailed in shear. In contrast, when the base shear coefficients of the frames were relatively large, i.e., $C_{0F} > 0{\cdot}5$ for the same ground motion records with $A_{max} = 0{\cdot}8g$, the response gradually decreased as the number of walls increased, whether or not the walls failed.

By normalizing the base shear coefficients of the frames and walls in terms of the peak ground accelerations, an empirical formula for the required base shear coefficients of the frames and walls to maintain a maximum displacement response for the frame structures within certain target values was derived for the Hachinohe and Tohoku University accelerograms. When the strength ratios of the frames were greater than those of the walls, a circular relation, equation (3), was found; whereas, the straight line relationship of equation (4) expressed the constant total strength ratios of the frames and walls for all the other cases. The same expressions held for the Fukiai and Kobe J. M. A. accelerograms when the magnitude of the required shear strength ratios was reduced by 15–20 per cent. These relations were derived for cases in which the flexural deformation of the walls always remained elastic, giving the upper bounds of the total strength ratios of the frames and walls. These ratios may prove to be of great practical importance in the design of short- to medium-height frame–wall-type buildings, as well as in retrofit planning for damaged buildings.

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